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# LIME SLURRY INJECTION, LIME PILES AND STONE COLUMNS FOR IMPROVEMENT OF SOFT SOILS - FIELD TRIALS

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## ABSTRACT:

Scarcity of good land available for construction has resulted in development of a number of techniques for improvement of ground. If properly treated, most soils can be made into useful construction materials. Two case histories are presented utilizing two different kinds of ground improvement techniques. Field scale trials are conducted to reinforce deep deposit of soft marine clay with stone columns and a road embankment made with black cotton soil modified with lime columns and pressure injection of lime slurry. Both techniques resulted in significant improvement in strength and settlement characteristics.

## KEYWORDS

Soft soils, ground improvement, lime slurry, lime piles, stone columns, bearing capacity, shear strength, stability analysis

## INTRODUCTION

Coastal areas of India have deep deposits of soft compressible marine clay. The depth of clay deposits vary from 6m to 18m. Undrained shear strength varies from $25kN/m^2$ to $40kN/m^2$. Permeability of the clay is low resulting in high consolidation time. Foundation of structures on such soil invariably require that the soil be strengthened. A number of ground improvement techniques viz, preloading, preloading with drains, stone columns etc have been applied with significant advantage. The choice of technique applied mostly depended. on economy, time and requirement of gain in strength. Another problematic soil is black cotton characterised by its low bearing capacity, high swelling and shrinkage characteristics resulting in volume instability due to change in water content in dry and rainy seasons. Roads in black cotton soil areas, falling in coastal delta regions are highly distressed. Foundation soil as such need modification. Black cotton soil is most economically modified by treatment with lime

## GROUND IMPROVEMENT WITH STONE COLUMNS

Vishakhapatnam port trust in S.E. India export iron ore in bulk. The ore is stacked on 1000mx35m strips. The soft clay, at 30kN/m2 can support 3m iron ore. Export requirement warrants stacking height of 9m. As and when the ore height increases more than 3m, instability of the stacker and reclaimer takes place resulting in sinking and heave up of track areas, Fig.1. This results in disruption of movement of ore. As such, the soft clay needs to be adequately strengthened. Stability analysis for bearing and rotational failure showed that 1m dia stone columns spaced at about 3mc/c can support 9 m height of iron ore.

*Fig.1 Upheaval of the track due to overloading*

Table-1. Engineering properties of Soft Marine Clay at Vishakhapatnam Port Area.

| | |
|---|---|
| 1. Depth of soft clay | 8m - 18m |
| 2. Water table | ground level |
| 3. Initial void ratio | 1.9 - 2.3 |
| 4.. Average unit wt | 1.5kN/$m^3$ |
| 5. Liquid Limit | 65 - 97% |
| 6. Plastic Limit | 24 - 55% |
| 7. Comp. Index Cc | 0.82 - 0.88 |
| 8. Coef. of Consl. Cv | 0.86x$10^{-5}$ m/day |
| 9. Coef. of Consl. Cr | 0.98x$10^{-3}$m/day |
| 10. Kv | 1.02x$10^{-5}$m/day |
| 11. Kh | 1.05x$10^{-5}$ m/day |

## SOIL PROFILE AND INDEX PROPERTIES

Insitu strength profile and material characteristics were determined by field van shear tests and undisturbed samples recovered from boreholes. The strength profile of the soft clay is shown in Fig.2. The engineering properties of the marine clay are given in Table-1.

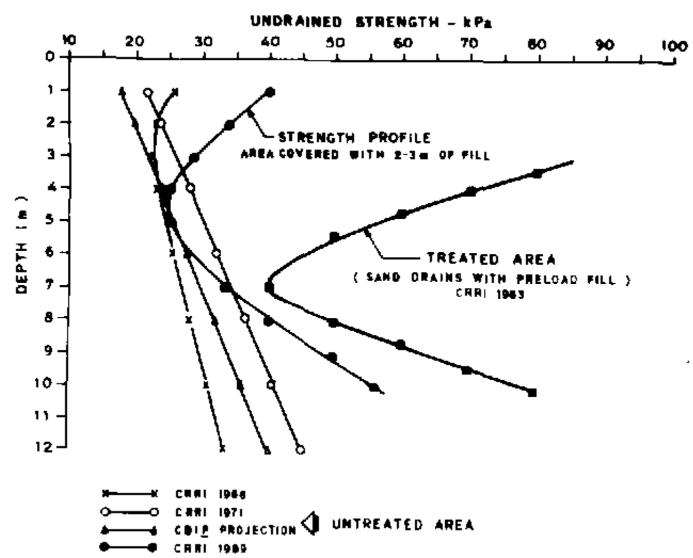


*Fig.2 Average shear strength profile in the area*

## STABILITY ANALYSIS

Stability analysis for bearing capacity and rotational failure on untreated soil due to loading by iron ore were carried out considering total stress analysis which gives stability under shear strength prevailing immediately after construction. The parameters used in the analysis are given in Table-2.

Table 2. Parameters used for stability analysis

| | |
|---|---|
| Depth of soft clay | 12m |
| Average Cu | 30kN/$m^2$ |
| Unit weight, clay | 15kN/$m^3$ |
| Water Table | ground level |
| Thickness of sand on soft clay | 2m |
| Unit weight, sand | 20kN/$m^3$ |
| Cohesion, sand | 0 |
| $\phi$ sand | $35^{\circ}$ |
| Unit wt, ore | 27.2kN/$m^3$ |
| cohesion, ore | 0 |
| $\phi$ ore | $33^{\circ}$ |

failure consideration Bearing capacity and rotational

3m height of iron ore and 2m height of sand produces a stress of 101.6kN/m2. With average Cu as 30kN/$m^2$, ultimate bearing capacity qult = 154kN/$m^2$. Factor of safety =154/101.6=1.48 which may be considered adequate for embankment type of fills. Stability analysis for 4m height of ore stack for rotational failure is shown in Fig. 3. Results indicate that the minimum factor of safety is for circles passing through a depth of about 4m below the sand layer even with assumed average value of 30kN/m2.

## NEED FOR GROUND IMPROVEMENT AND INSTALLATION OF STONE COLUMNS

Results indicate that the soft marine clay is unable to support an ore height exceeding 4m. Since the requirement is to stack ore upto a height of 9m and the treatment should afford placement of 9m of ore immediately. stone column technique was recommended. Due to limited funds available, only 19 stone columns could be installed. The configuration of stone columns with type of test are shown in Fig.4. The grading of backfill material used in stone columns is given in table 3.

Table 3. Gradation of material used in stone columns

| Aggregate size mm | %finer | Aggregate size mm | %finer |
|---|---|---|---|
| 75 | 90-100 | 20 | 10-20 |
| 50 | 80-90 | 12 | 5-13 |
| 38 | 55-75 | 2 | $\angle$5 |

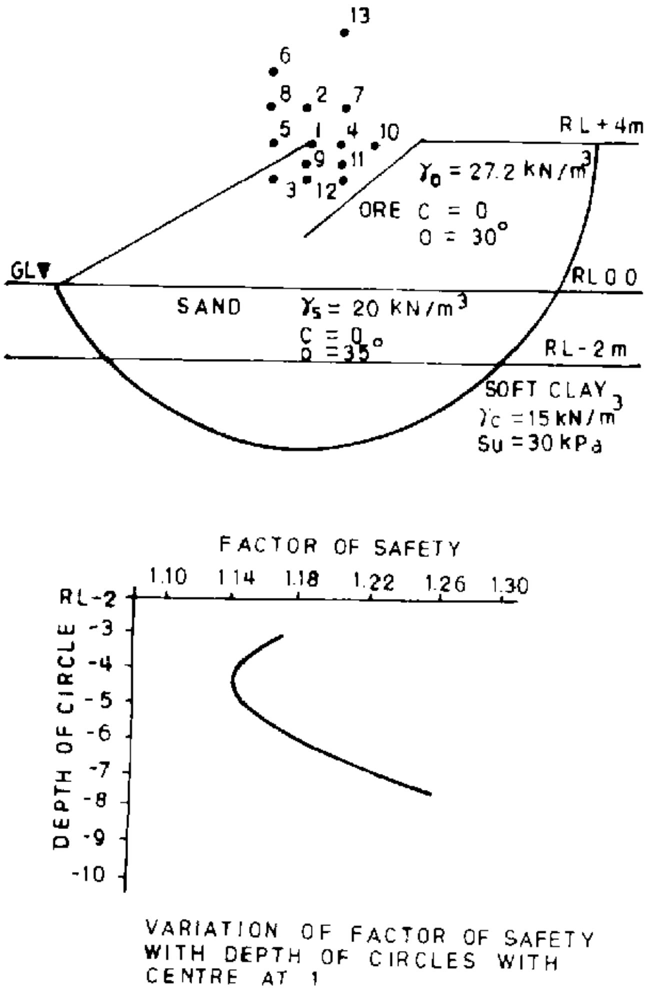


*Fig.3. Stability Analysis for 4m Height of Ore on untreated soil*

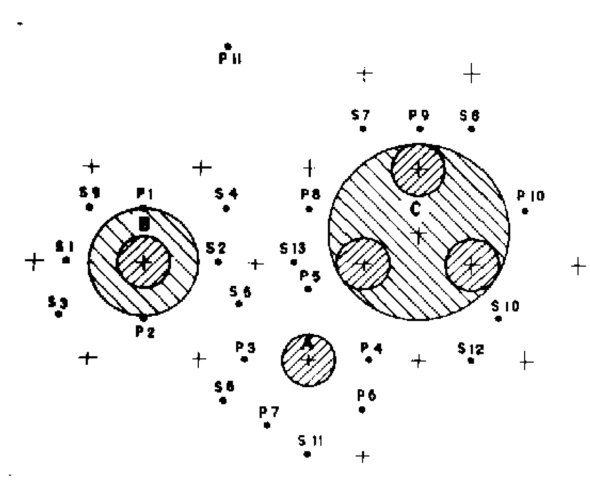


*Fig. 4. Stone Column Test Locations*

The test columns were arranged so as to have maximum number of columns around each test column. The columns were made of 1m dia at 2m c/c in triangular pattern with rammed column technique. The length of column was 18m resting on a firm base. Estimated ultimate bearing capacity of stone columns with different methods is shown in Table-4.

Table 4. Calculated ultimate load bearing capacity of stone columns

| Method of Computation | Authors | Ultimate load cap. kN |
|---|---|---|
| Limit lateral stress | Hughes & Withers, 1974 | 520 |
| Cavity Expansion | Vesic, 1972 | 735 |
| Empirical | Datye, 1982 | 1180 |
| Unit Cell | Nayak, 1982 | 1120 |

Table-4 shows that there is a large variation in the computed load capacities of stone columns by different theoretical and empirical approaches. Therefore, field load tests were undertaken as per the load capacities shown in Table 5. TheTable-4 also shows recorded settlements under the loads. The results of the load tests are shown in Fig.5.

Table-5. Results of three load tests

| Type of test | Max. Load kN | Observed settlement |
|---|---|---|
| Single Column | 1000 | 9.5mm |
| Unit Cell | 1800 | 20.0mm |
| Three columns | 2000 | 3.0mm |

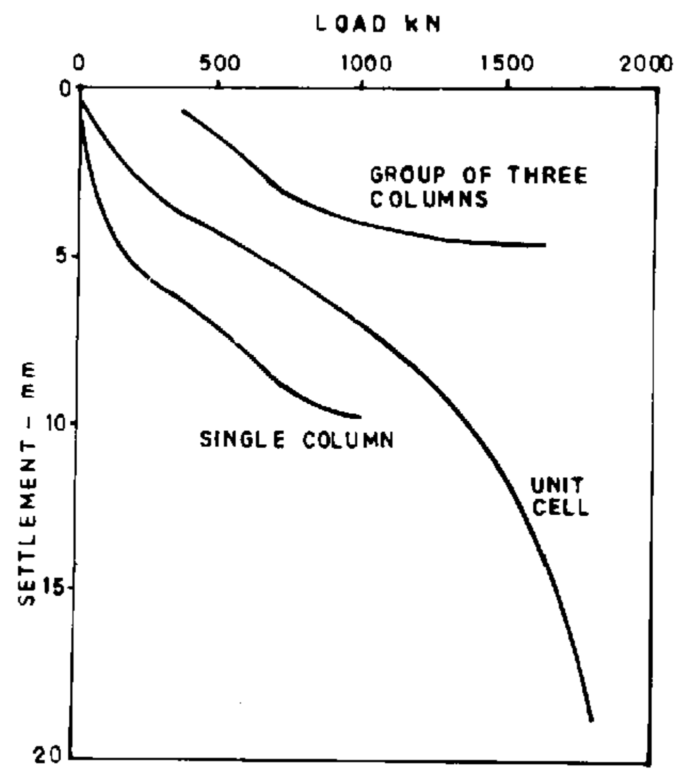


*Fig.5. Load test results*

## STABILITY ANALYSIS OF REINFORCED GROUND

Stability analysis was done in the critical state. Average unit weight and average shear strength along the failure plane within the unit cell were evaluated and used in stability analysis. The columns were assumed to extend only upto the toe of the stack. analysis was carried out for different heights of ore stack.. Two cases for sequence of laying of stockpile were considered while analysing the stability:(i) two stage construction, 0m-4m and 4m-9m and (ii) three stage construction with a lift thickness of 3m each allowing the desired waiting period of 3 months between each stage of loading. When the ore height is increased in stages with appropriate waiting period, the increase due to the consolidation phenomenon was found according to the equation $Su = 0.18\ \ p$, where Su is gain in shear strength. The results of the stability analysis are given in Figs. 6, 7

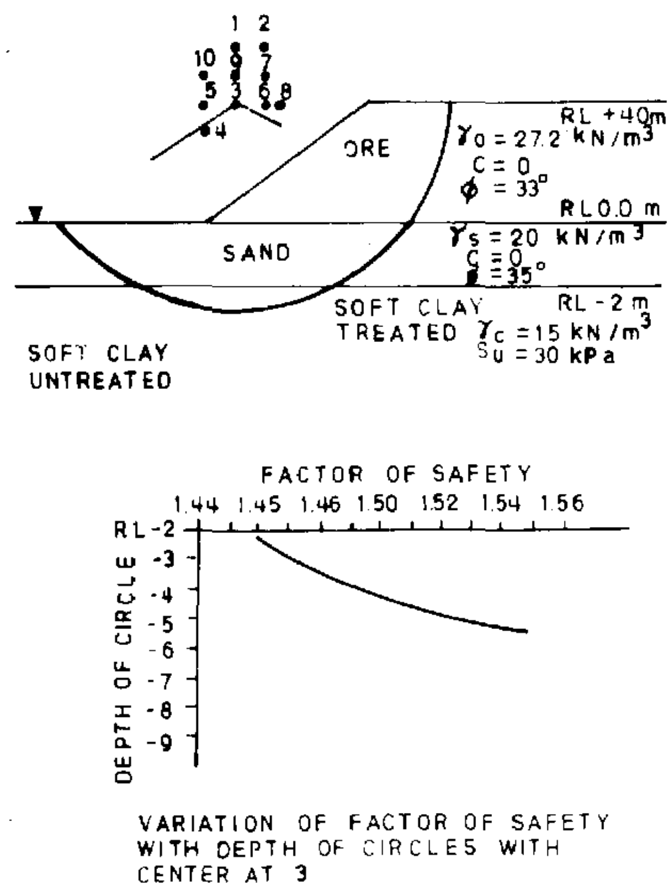


*Fig.6. Stability Analysis for 4m Height of Ore Stack on Treated Soil.*

## INTERPRETATION OF TEST RESULTS

Maximum settlement observed was of the order of 20mm under a load of 18000 kN in the case of unit cell test. This is anticipated as the loading plate is of 2.1m dia and the area replacement ratio is 0.23 indicating more of soft clay under the loading plate which is of low bearing capacity. The stress is 243kN/m2 due to iron ore compared to a stress intensity of 520kN/m2 which a unit cell can sustain safely. Simple calculation can show that spacing can be increased to approximately 3 mc/c. The unit cell can safely sustain a load of 1720kN/m2.

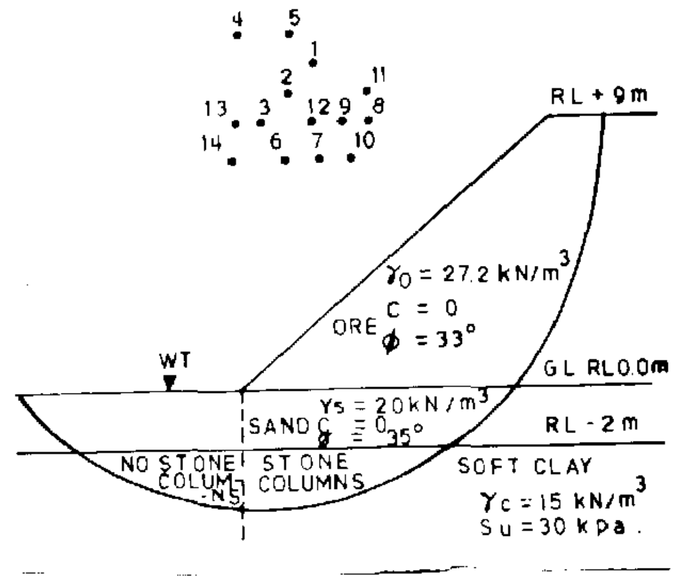


*Fig.7. Stability Analysis for 9m Height of Ore Stack on Treated Soil.*

## GROUND IMPROVEMENT BY LIME PILES AND LIME SLURRY INJECTION

Construction of roads in the black cotton soil areas has been problematic due to their low bearing capacity and the detrimental effects of volume instability arising from swelling and shrinkage characteristics of such soils during wet and dry seasons. Such problems assume greater acuteness where water table is high and clayey subsoil stratum is thick. A number of road embankments in the delta region of rivers Godavari and Krishana have been experiencing distress for the past many years, One of the road surveyed is Gudivada-Bantumilli road which is a state highway. The road passes through plain terrain covered with paddy fields and the same is constructed on a black cotton soil embankment of approximately 3m height. The pavement on the embankment is in a state of extreme distress. Lateral shift and settlement of the embankment are common features observed.

To verify the efficacy of the technique, laboratory experiments were conducted at Central Road Research Institute on a test embankment of 2.5mx2.0mx2.5m made from the locally available silty soil having a dry density of 16 kN/m$^2$. Lime slurry was injected in one part of the embankment. The injected lime contained 30% lime by weight in the mixture of lime-water. The injection needle was pushed in the soil at appropriate depth and slurry injected at about 400kN/m$^2$ pressure. The radial migration of lime and lime content at various radial distances for depths of injection of 1m and 1.5m are shown in fig.8.

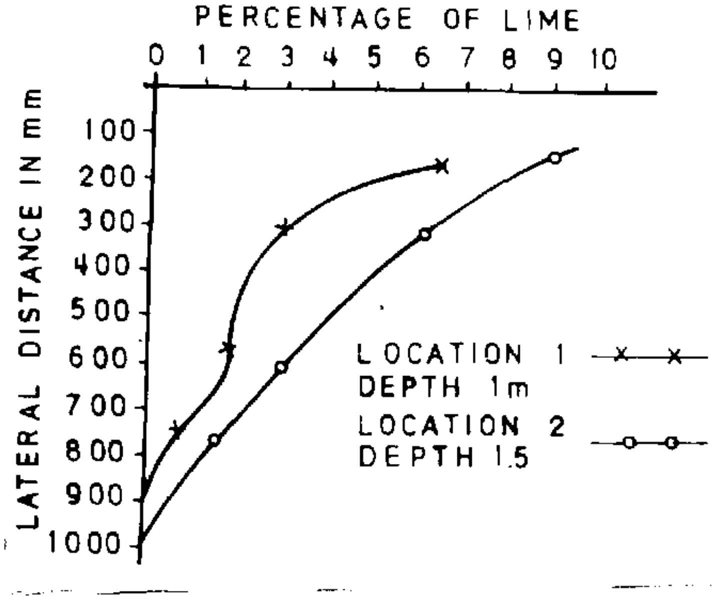


*Fig.8. Lateral Distribution of Lime Slurry*

In the second part, eight lime columns were made. The columns were 100mm in dia and 2.5m deep and were made with lime concentration of 10%, 15% and 20% and 25% by weight of soil. Two columns were made for each of the chosen percentages of lime. Plate load tests were performed on each of these columns. Fig..9 shows results of the plate load tests after 28 days of curing period. It is evident that k-values increase from 370kN/cm$^2$ to 600kN/m$^2$ and the increase is maximum for lime content of 15% in the columns. Pressuremeter tests were also conducted and an increase of 100% in modulus of elasticity was observed. The limit pressure values also showed an increase of 50% due to modification by pressure injection of lime.

## EXECUTION OF THE FIELD EXPERIMENTAL PROGRAMME

### Lime Columns

A programme to execute the strengthening of the embankment was formulated using (a) lime piles (b) Lime slurry injection (c) Lime fly-ash slurry pressure injection. Lime piles were installed upto a depth of 4m and at a spacing of 2m c/c in an equilateral triangular pattern in two rows having a total of 62 piles, Fig. 10. The treatment was confined to 50m shoulder region. The boreholes were made by displacement method. The backfill material contained 15% lime by weight of the soil, thoroughly pulverized and brought to OMC and compacted in the boreholes in small vertical shifts forming columns.

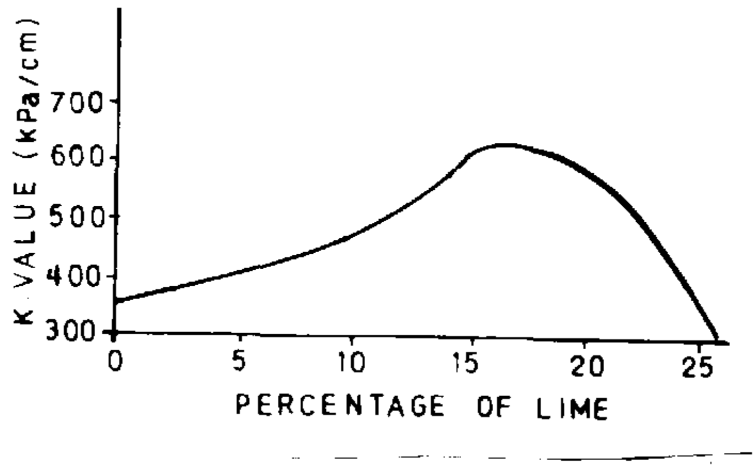


*Fig 9. Plate Load Tests on Lime Columns*

### Lime slurry injection

A stretch of 50m adjacent to the area treated with lime piles was used for lime slurry injection at a spacing of 1.5m c/c in a triangular pattern. The depth of injection was 4m and upwards at a vertical spacing of 1m each. 62 points, as shown in the Fig.11 were treated with lime slurry injection at a pressure of 1300kN/m$^2$ to 1400kN/m$^2$. 30% lime by weight was added to water to prepare lime slurry. In each borehole, 200 liters of slurry was injected at each borehole without any sign of refusal. In some cases, fly ash was added to lime slurry in the ratio of 1:1 lime:flyash). The mixture caused clogging of the pressure pump making the system immobile. Subsequently, slurry made in the ratio of 1:0.5 was prepared and did not cause any clogging of the pressure pump.

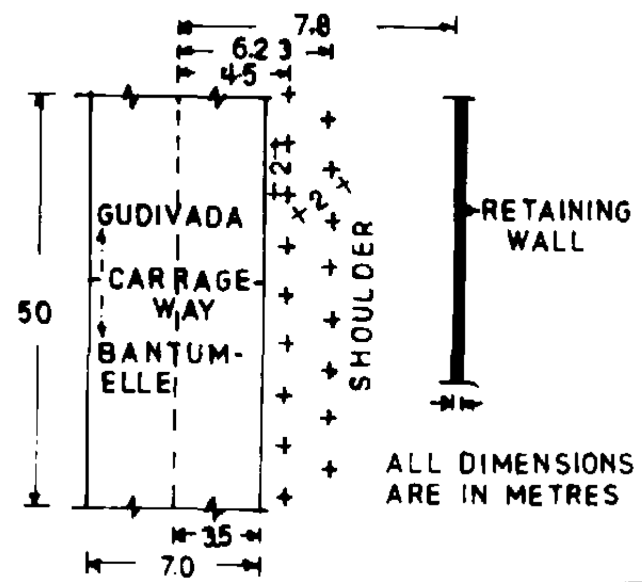


*Fig 10. Location of Lime Piles*

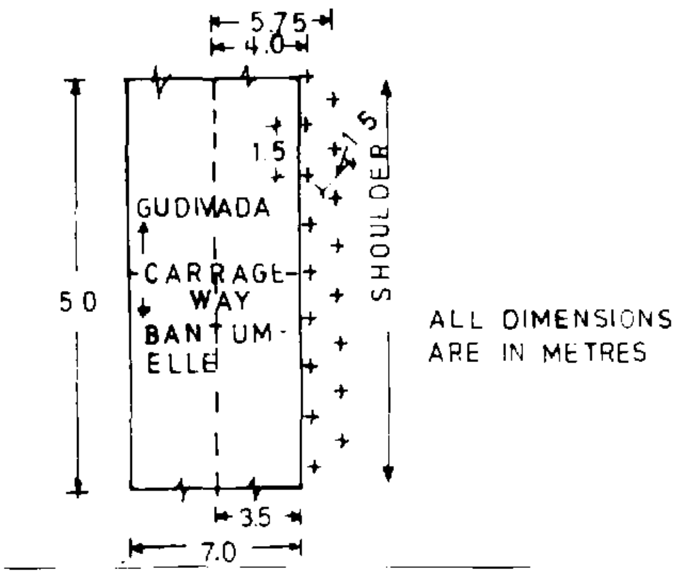


*Fig.11. Points of Lime Slurry Injection.*

## OBSERVATIONS

Five to six lime piles could be installed by manual operation in a day by four skilled men. Pressure injection lime slurry was done by a portable equipment fabricated at CRRI. Although, lime slurry injection did not met with any refusal, in some cases, slurry started coming out within 2m region from injection points or through cracks and crevices in the soil. No problem was encountered in the pressure injection even under the water table.

The observations reveal the presence of shrinkage cracks in the embankment soil which may be the source of distress to the pavement. Lime slurry injected is bound to fill them resulting in prevention of percolation of rain water and provide adequate strength to the embankment soil. The same pattern was also observed by the pressure meter test results which were performed before and after the treatment. An increase of 150% and i100% increase in the elastic modulus was observed in the area treated with lime slurry and lime piles respectively after a period of about 1year. Subsequent to the treatment, no lateral shift or pavement settlement was observed in the treated portion.

## CONCLUSIONS

Appropriate ground improvement techniques can be applied to strengthen existing weak deposits of soils to achieve desired improvement in bearing and settlement of foundation soil. Due to lack of confidence in various available theoretical or empirical formulations to compute bearing and settlement of treated foundations, it is advisable to evaluate the performance with field trials and related test programme.



## ACKNOWLEDGEMENT

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